Environmental Engineering

Index

Water Demand & sources of water

(i) As per GO Fire Demand

$=100(P)^{1/2}$

(ii) Kuichling's Formula

$$\boxed{Q = 3182\sqrt{P}}$$

Where Q = Amount of water required in liters/minute.

P = Population in thousand.

(iii) Freeman Formula

$$\boxed{Q = 1136\left[\frac{P}{10} + 10\right]}$$

(iv) National Board of Fire Under Writers Formula

(a) For a central congested high valued city

(i) Where population < 200000

$$\boxed{Q = 4637\sqrt{P}\left[1 - 0.01\sqrt{P}\right]}$$

(ii) where population > 200000

Q = 54600 lit/minute for first fire

and Q=9100 to 36,400 lit/minute for a second fire.

(b) For a residential city.

(i) Small or low building,

Q=2,200 lit/minutes.

(ii) Larger or higher buildings,

Q=4500 lit/minute.

(iii) High value residences, apartments, tenements

Q=7650 to 13,500 lit/minute.

(iv) Three storeyed buildings in density built-up sections,

Q=27000 lit/minute.

(iv) Buston's Formula

$$\boxed{Q = 5663\sqrt{P}}$$

The probability of occurrence of a fire, which, in turn, depends upon the type of the city served, has been taken into consideration in developing the above formula on the basis of actual water consumption in fire fighting for Jabalpur city of India. The formula is given as

$$\boxed{Q = \frac{4360R^{0.275}}{(t+12)^{0.757}}}$$

Where,

R = Recurrence interval of fire i.e., period of occurrence of fire in years, which will be different for residential, commercial, and industrial cities.

Per Capita Demand (q)

$$\boxed{q = \frac{Total\ yearly\ water\ requirement\ of\ the\ city\ in\ litre}{365 \times Design\ population}}$$

Assessment of Normal Variation

(i) $\boxed{Maximum\ daily\ deman = 1.8 \times Avg\ daily\ demand}$

(ii) $\boxed{Maximum\ hourly\ demand = 1.5 \times Avg.\ demand\ hourly\ of\ max.\ day}$

(iii) $\boxed{Maximum\ hourly\ demand\ or\ peak\ demand = 2.7 \times Avg\ Hourly\ demand}$

(iv) $\boxed{\frac{Maximum\ daily\ demand}{Avg\ daily\ demand} = 180\%}$

(v) $\boxed{\frac{Maximum\ weekly\ demand}{Avg\ weekly\ demand} = 148\%}$

(vi) $\boxed{\dfrac{Maximum\ monthly\ demand}{Avg\ monthly\ demand} = 128\%}$

Population forecasting Methods

(i) Arithmetic increase method

$\boxed{P_n = P_0 + n\bar{x}}$

Where,

Prospective or forecasted population after n decades from the present (i.e., last known census)

Population at present (i.e., last known census)

$n =$

Number of decades between now & future.

Average (arithmetic mean) of population increases in the known decades.

(ii) Geometric Increase Method

$\boxed{P_n = P_o\left(1 + \dfrac{r}{100}\right)^n}$

where,

$P_o =$ Initial population.

$P_n =$ Future population after ‘n’ decades.

$r =$ Assumed growth rate (%).

$\boxed{r = \sqrt[t]{\dfrac{P_2}{P_1}} - 1}$

where,

$P_2 =$ Final known population

$P_1 =$ Initial known population

$t =$ Number of decades (period) between P_1 and P_2.

$$r = \sqrt[t]{r_1 \, r_2 \ldots r_t}$$

(iii) Incremental Increases Method

$$P_n = P_o + n\overline{x} + \frac{n\left(n=1\right)}{2}\overline{y}$$

Where,

$\overline{x}$ = Average increase of population of known decades

$\overline{y}$ = Average of incremental increases of the known decades.

(iv) Decreasing rate of growth method

Since the rate of increase in the population goes on reducing, as the cities reach towards saturation, a method which makes use of the decrease in the percentage increase, in many times used, and gives quite rational results. In this method, the average decrease in the percentage increase is worked out, and is then subtracted from the latest percentage increase for each successive decade. This method is, however, applicable only in cases, where the rate of growth shows a downward trend.

(v) Logistic Curve Method

(a) $$\log_e\left(\frac{P_s - P}{P}\right) - \log_e\left(\frac{P_s - P_o}{P_o}\right) = -k\,P_s\,t$$

Where,

P_o = Population of the start point.

P_s = Saturation population

P = Population at any time t from the origin.

k = Constant.

(b) $$P = \frac{P_s}{1 + m\log_e^{-1}\left(nt\right)}$$

(c) $$P_s = \frac{2P_0P_1P_2 - P_1^2\left(P_0 + P_2\right)}{P_0P_2 - P_1^2}$$

(d) $\boxed{m = \frac{P_s - P_o}{P_o}}$

(e) $\boxed{n = \left(\frac{1}{t_1}\right)\log_e\left[\frac{P_o(P_s - P_1)}{P_1(P_s - P_o)}\right]}$

Quality of water

Water Quality Parameters

Total Solid and Suspended Solid

$$\boxed{\text{Total solid} - \text{Suspended solid} = \text{Dissolved solid}}$$

Acceptable limit of total solid = 500 mg/lit

Threshold odour number

$$\boxed{T.O.N = \frac{\text{Final volume at which odour is hardly detectable}}{\text{Sample volume}}}$$

where TON = Threshold odor number

$\boxed{1 \le TON \le 3}$

constituents of Alkalinity

Major sources $\rightarrow CO_3^{2-}, HCO_3^-, OH^-$

Minor sources $\rightarrow HS_iO_3^-, H_2BO_3^-, HPO_4^{2-}, HS^-$

(a)

$$\boxed{\text{Equivalent weight} = \frac{\text{Molecular weight}}{\text{valency}}}$$

Equivalent weight of

$$CaCO_3 = \frac{40 + 12 + 3 \times 16}{2} = 50$$

(b)

$$\boxed{\text{gram equivalent or number of equivalent} = \frac{\text{weight}}{\text{equivalent weight}}}$$

$$Ca^{2+} \quad + \quad 2CI^- \quad \rightarrow \quad CaCI_2$$

$1\,mole \qquad 2\,mole \qquad 1\,mole$

$1\,equivalent \quad 1\,equivalent \quad 1\,equivalent$

$$\begin{aligned} equivalent\ of\ Ca^{2+} & \\ &= equivalent\ of\ CI^- \\ &= equivalent\ of\ CaCI_2 \end{aligned}$$

(c)

$$\boxed{Alkalinity\ of\ water = (x + y + z)\, mg\,/\,lit\ of\ CaCO_3}$$

where,

$CO_3^{2-} \rightarrow x\, mg\,/\,lit\ of\ CaCO_3$

$HCO_3^- \rightarrow y\, mg\,/\,lit\ of\ CaCO_3$

$OH^- \rightarrow z\ mg\,/\,lit\ of\ CaCO_3$

Value of water

$$\boxed{pH = -\log_{10}[H^+]}$$

$$\boxed{pH + pOH = 14}$$

$$\boxed{[H^+] > 10^{-7}}$$

For strong acid.

$$\boxed{pOH = -\log_{10}[OH^-]}$$

$$\boxed{[H^+][OH^-] = 10^{-14}}$$

$$\boxed{[H^+] < 10^{-7}}$$

For strong base

$$\boxed{[H^+] = [OH^-] = 10^{-7}}$$

For neutral solution

Hardness of water

$$\text{Total Hardness} = \left[\left\{\frac{[Ca^{2+}]}{\text{Equivalent weight of } Ca^{2+}}\right\} \times \text{Equivalent weight of } CaCO_3\right] + \left[\left\{\frac{[Mg^{2+}]}{\text{Equivalent weight of } Mg^{2+}}\right\} \times \text{Equivalent weight of } CaCO_3\right]$$

where,

$[Ca^{2+}]$ *is concentration of* Ca^{2+} *in mg / lit.*

$[Mg^{2+}]$ *is concentration of* Mg^{2+} *in mg / lit*

(a)

$$\boxed{\text{Total hardness} = \left[\frac{[Ca^{2-}]}{20} \times 50 + \frac{[Mg^{2+}]}{12} \times 50\right] \text{ mg / lit as } CaCO_3}$$

Where,

atomic weight of Ca = 40

atomic weight of Mg = 24

atomic weight of O = 16

atomic weight C = 12

valency of Ca = 2

Valency of Mg = 2

$$\text{Equivalent weight of } Ca^{2+} = \frac{\text{atomic weight}}{\text{valency}} = \frac{40}{2} = 20$$

$$\text{Equivalent weight of } Mg^{2+} = \frac{\text{atomic weight}}{\text{valency}} = \frac{24}{2} = 12$$

$$Equivalent\ weight\ of\ CaCO_3$$
$$= \frac{Moleculasweight}{valency}$$
$$= \frac{40 + 17 + (3 \times 16)}{2} = 50$$
$$As\ \ CaCO_3 \rightarrow Ca^2 = CO_3^{2-}$$
$$\therefore \quad Valency\ or\ n - factor = 2.$$

(b) $$\boxed{Carbonate\ Hardness = Minimum\ of \begin{cases} Total\ hardness \\ Alkalinity \end{cases}}$$

(c) $$\boxed{Non\ carbonate\ hardness = Total\ hardness - Alkalinity}$$

Note: -ve value is taken as zero.

(d) **Hardness**

$$0 - 55 \xrightarrow{mg/lit\ as\ CaCO_3} Soft$$
$$56 - 100 \xrightarrow{mg/lit\ as\ CaCO_3} Slightly\ hard$$
$$101 - 200 \xrightarrow{mg/lit\ as\ CaCO_3} Moderately\ hard$$
$$201 - 500 \xrightarrow{mg/lit\ as\ CaCO_3} Very\ hard$$

Household supplies limit 74-115 mg/l

(e) The acceptable limit of hardness = 200 mg/lit as

and cause of rejection is 600 mg/lit as $CaCO_3$

Biochemical Oxygen Demand (BOD)

$$\boxed{(BOD)_5 = (DO_i - DO_f) \times Dilution\ Ratio}$$

whereof 5 days

$DO_i - DO_f =$ Loss of oxygen in mg/lit.

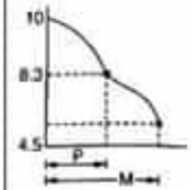
$DO_i =$ Initial dissolved oxygen concentration in the diluted sample.

$DO_f =$ Final dissolved oxygen concentration in the diluted sample.

(After 5 day of incubation of 20^{o}C)

Water Quality Parameters	Instrument used	Additional information
1. Turbidity	1. Turbidity Rod (Absorption Principle) Field Method	
	2. Jackson Tubidmeter method (Absorption Principle Haboratory method	(i) Only for sample with turbidity > 25 units (ii) Generally for water in natural bodies and hot for water supply
	3. Baylis Turbidmeter (Absorption) (colour matching echnique)	(i) Can measure Turbidity < 1unit
	4. Nephlometer	(i) Can measure Turbidity < 1 unit
2. Colour	Tintometer (using Nessler tubes)	Std. unit TCU Pt in (chloroplatinate from) in 1 l of water
3. Taste and Odour	Osmoscope	Std. unit TON

- Turbidity>5 units–detectable by naked eye.
- In Nephlometer, turbidity is expressed in FTU.

Chemical W.Q.P.	Measurement	
1. Alkalinity	Titratant : 0.02 N H2SO4 Indicates : Phenolphthaline (Basic Indicator) Methyl Orange (acidic indicator)	$OH^- + H^+ \rightarrow H_2O$ $CO_3^{2-} + H^+ \rightarrow HCO_3^-$  (i) $P < \frac{M}{2}$ (Most common observation)
1. PH	Instrument : Acquascope Potentiometer (with Calomel Rods)	
3. Hardness	Titration Titiant : 0.01 MEDTA	British Degree of Hardness = 14.25 mg/l (as $CaCO_3$) French D.O.H. =10 mg/l (as $CaCO_3$)
4. Dissolved	i. Graviometric Technique ii. Di-ionic tester	Electrical conductivity calliberated against total dissolved solids
5. Chloride	Gitrant : $AgNO_3$ solution Indicator : $K_2C_rO_4$	
6. Nitrogen content	i. Free $NH_3 \rightarrow$ Simple boiling and distribution ii. Organic : Addition of $KMnO_4$ to already boiled water sample iii. Nitrite : Colour making (Sulphonic acid and Naptha-mine) iv. Nitrate : Phenol disulphonic + KOH acid	INDICATE Recent Pollution N_2 before decomposition
7. Fluoride content	Colour Matching Colour indicued by zirconium	If<1 ppm – Dental cavities If>1.5 ppm –Fluorosis
8. Oxygen	Winkler's Method	If nitrite ion present. Modified Winkler's adopted

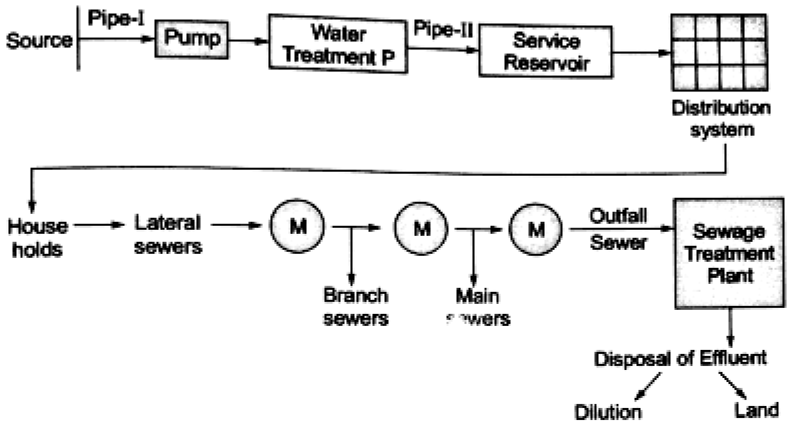
Biological WQP

Testing of Colliforms is done

Tests/Technique	Remarks
1. MPN Test	i. Multiple tube Feimentation test ii. Nutrient used : Lactose broth iii. More is the dilution of sample lesser is the possibility of getting +ve test.
2. Membrance Filter Technique	i. Nutrient : M-Endo Medium ii. Colliform colonies are counted
3. Colliform Index Test	Reciprocal of smallest quantity of sample giving +ve B-coil test

	Water Quantity parameter	Permissible Units	Cause for Rejection Value
1.	Suspended solid	500 mg/l	2000 mg/l
2.	Turbidity	1 NTU	10 NTU
3.	Colour	5 TCU	25 TCU
4.	Taste and Odour	1 TON	3 TON
5.	Alkalinity	200 mg/l	600 mg/l
6.	pH	7 – 8.5	<6.5
7.	Hardness	200 mg/l	600 mg/L
8.	Chloride content	200 mg/l	1000 mg/L
9.	Fre NH3	0.15 mg/l	0.15 mg/l
	Organic NH3	0.3 mg/l	0.3 mg/l
	Nitrite	0	0
	Nitrate	45 mg//	
10.	Fluoride	1 mg/l	1.5 mg/l
11.	Fe	0.1 mg/l	1 mg/l
12.	Mn	0.05 mg/l	0.5 mg/l
13.	Cu	0.05 mg/l	1.5 mg/l

Treatment of raw water



Theory of Sedimentation

- **Stokes Law**

(a)

$\boxed{V_s = \frac{g}{18}\left(G-1\right)\frac{d^2}{v}}$ for d < 0.1 mm.

Where,

$V_s =$ The velocity of the settlement of particle in m/s.

The diameter of the particle in the meter.

$G =$ SP gravity of the particle

$= \frac{\gamma_s}{\gamma_w}$ or $\frac{\delta_s}{\delta_w}$

$v =$ Kinematic viscosity of water in m^2/sec.

(b)

$$\boxed{V_s = \left[\frac{\frac{4}{3}gd\left(G-1\right)}{C_D}\right]^{1/2}}$$

$\boxed{C_D = 0.4} \rightarrow For\left(R_e > 10^4\right)$

$\boxed{C_D = \frac{24}{R_e}} \rightarrow For\left(R_e < 0.5\right)$

$$\boxed{C_D = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + 0.34} \; For \; \boxed{0.5 \le R_e \le 104}$$

(c)

$$\boxed{V_s = 418\,(G-1)\,d^2\left(\frac{3T+70}{100}\right)} \; for \; d < 0.1\,mm$$

Where,

T = Temperature of water in ^{o}C

V_s is in mm/sec.

d is in mm.

(d) $\boxed{V_s = 1.8\sqrt{gd\,(G-1)}}$ For $d > 0.1\,mm.$

(e) $\boxed{V_s = 418\,(G-1)\,d\left(\frac{3T+70}{100}\right)}$ For $0.1 < d < 1\,mm.$

Sedimentation Tank : Surface Over flow Rate

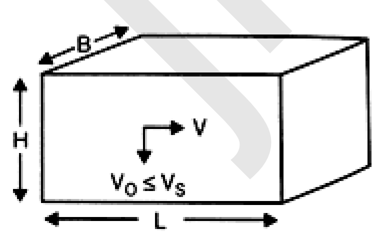
$$\boxed{Over\ flow\ rate,\ V_o = \frac{Discharge}{Surface\ area}}$$

$$\downarrow$$

(a) $\boxed{V_o = \frac{Q}{BL}}$

12000 to 18000 lit/m2 day for plain sedimentation.

$V_o =$ 24000 to 30,000 lit/m2/day for sedimentation with coagulation.



(b) Velocity of flow, $\boxed{V_f = \frac{Q}{BH}}$

(c) Time of horizontal flow, or Detention time

$$\boxed{T = \frac{L}{V_f} = \frac{L}{Q / BH} = \frac{LBH}{Q}}$$

(d) Time of falling through height 'H'

$$\boxed{T = \frac{H}{V_s} = \frac{LBH}{Q}}$$

(e) Detention time,

$$\boxed{t_d = \frac{L}{V_f} = \frac{H}{V_s}}$$

For plain sedimentation tank → 4 to 8 hours

For sedimentation with coagulation → 2 to 4 hours

(f) η, Efficiency

$$\boxed{P_e = \frac{V_s}{V_o} \times 100}$$

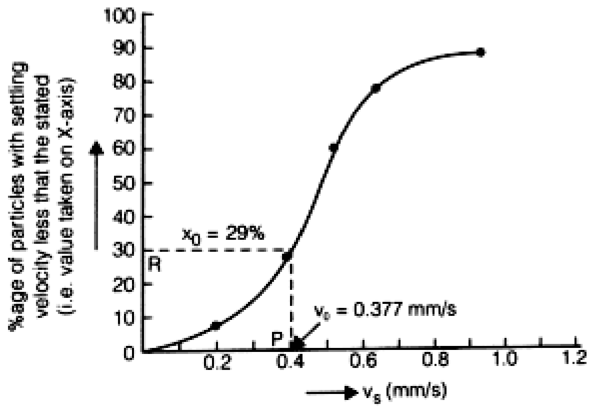
where,

$P_e = \%$ of lighter particles (with settling velocity (V_s)less than V_o) which shall be removed in an ideal settling basin.

(g) % of particle removed

$$= (100 - x_0) + \int_{x=0}^{x=x_0} \left(\frac{V_s}{V_o} \times 100 \right) \cdot dx$$

where corresponds to V_0



(h) Detention time 't'

$\boxed{t = \frac{BLH}{Q}}$ for a rectangular tank.

$\boxed{t = \frac{d^2\left(0.011d + 0.785H\right)}{Q}}$ for circular tank

where,

d = Dia. of the tank

H = Vertical depth of wall or side water depth

(i) $\boxed{Displacement\ efficiency = \frac{Flowing\ through\ period}{Detention\ \ period}}$

(j) Scour velocity, V_d $\boxed{V_d = \sqrt{\frac{8\beta}{f\,'} g\left(G-1\right)d}}$

Where,

$\beta =$ 0.04 for uniform granular sand and 0.06 or more for non-uniform (interlocking) sticky material.

$f\,' =$ Darcy Weisbach friction factor.

= 0.025 to0.03 for settling tanks.

Chemicals used for Coagulation

- Alum $\left(Al_2\left(SO_4\right)_3.18H_2O\right)$

$$Al_2(SO_4)_3.18H_2O + 3Ca(HCO_3)_2 \rightarrow 3CaSO_4$$
$$+2Al(OH)_3 \downarrow + 6\,CO_2 \uparrow$$

666 gm of Alum reacts with 48 gm of to give 146 gm of $Al(OH)_3$

$\Rightarrow$ 1 gm of Alum reacts with 0.73 gm of $Ca(HCO_3)_2$ alk. or 0.45 gm of $CaCO_3$ alk.

to give 0.234 gm of $Al(OH)_3$ ppt.

$$Al_2(SO_4)_3.18H_2O + 3Ca(OH)_2 \rightarrow 3CaSO_4 +$$
$$Al(OH)_3 \downarrow + 18H_2O$$
$$Al_2(SO_4)_3.18H_2O + 3Na_2CO_3 \rightarrow 3Na_2SO_4 +$$
$$2Al(OH)_3 \downarrow + 3CO_2 \uparrow + 15H_2O$$

- Copperas $(FeSO_4.7H_2O)$

$$\underset{Copperas}{FeSO_4.7H_2O} + \underset{Hydrated\ line}{Ca(OH)_2} \rightarrow CaSO_4 + \underset{Ferrous\ hydroxide}{Fe(OH)_2} + 7H_2O$$
$$FeSO_4.7H_2O + Ca(HCO_3)_2 \rightarrow Fe(HCO_3)_2 + CaCO_3 + 7H_2O$$
$$Fe(HCO_3)_2 + 2Ca(OH)_2 \rightarrow Fe(OH)_2 + 2CaCO_3 + 2H_2O$$
$$Fe(OH)_2 + O_2 + 2H_2O \rightarrow \underset{Ferric\ Hydroxide}{4Fe(OH)_3 \downarrow}$$

- Chlorinated Copperas : $(Fe_2(SO_4)_3\ and\ FeCl_3)$

$$6(FeSO_4.7H_2O) + 3Cl_2 \rightarrow 2Fe_2(SO_4)_3 + 2FeCl_3$$
$$+42H_2O$$
$$Ferric\ sulphate$$
$$\underset{Ferric\ Sulphate}{Fe_2(SO_4)_3} + \underset{Hydrated\ line}{3Ca(OH)_2} \rightarrow 3CaSO_4 + \underset{Ferric\ hydroxide\ ppt}{2Fe(OH)_3 \downarrow}$$
$$\underset{Ferric\ Chloride}{2FeCl_3} + \underset{Hydrated\ line}{3Ca(OH)_2} \rightarrow 3CaCl_2 + \underset{Ferric\ hydroxide\ ppt}{2Fe(OH)_3 \downarrow}$$

- Sodium Aluminate

$$Na_2Al_2O_4 + Ca(HCO_3)_2 \rightarrow CaAl_2O_4 \downarrow + Na_2CO_3$$
$$+ CO_2 \uparrow + H_2O$$
$$Na_2Al_2O_4 + CaAl_2 \rightarrow CaAl_2O_4 \downarrow + 2NaCl$$
$$Na_2Al_2O_4 + CaAl_4 \rightarrow CaAl_2O_4 \downarrow + Na_2SO_4$$

Mixing Basin

Where,

G’ = Temporal mean Velocity gradient (per second).

P = Power dissipated in watts i.e., N-m/s.

V = Volume of raw water to which P is applied in m^3.

$\mu =$ Dynamic viscosity (N-s/m^2).

Flocculation

- Velocity gradient,

$$\boxed{G'\left[\frac{P}{\mu V}\right]^{1/2}}$$

- $20\ sec^{-1} < G' < 75\ sec^{-1}$.
- Detention time is 10 to 30 minutes.
- $$\boxed{G'.t_d\,(Conjunction\ opportunity) = \frac{Power\ induced\ rate\ of\ flow}{Displacement\ induced\ rate\ of\ flow}}$$
- Number of particle collision $\alpha\ G't_d$.
- $G'td = 2 \times 10^4\ to\ 6 \times 10^4\ for\ Alum.$
  $= 1 \times 10^5\ to\ 1.5 \times 10^5\ for\ Iron\ salt.$
- $$\boxed{\frac{G'\ of\ influent\ end}{G'\ of\ effluent\ end} = 2}$$

Filtration

A. Slow Sand Filter

- Depth of filter is 2.5 to 3.5 m.
- Plan area of the filter is 100 to 200 m^2.

- $0.2 \le D_{10}\ of\ \ sand \le 0.3\ mm.$
- $\frac{D_{60}}{D_{10}} = 5.$
- Design period = 10 years.
- Depth of sand is 90 to 110 cm.
- Frequency of cleaning is 1 to 3 months
- Rate of filtration = 2400 to 4800 lit/m^2/day or 100 to 200 lit/m^2/hr.
- Efficiency of bacteria removal = 98 to 99%.
- It can not be used if turbidity > 50 ppm.
- It is designed for maximum daily demand.
- $\boxed{\frac{Discharge}{Rate\ of\ \ filteration} = Plan\ \ area}$.

B. Rapid Sand Filter

- $\boxed{N = 1.22\sqrt{Q}}$

where,

N = Number of unit required

Q = Plant capacity in million lit/day (MLD)

- $\frac{D_{60}}{D_{10}} = 1.2\ \ to\ 1.8$
- Sand layer depth is 60 to 90 cm.
- D_{10} of sand is 0.35 to 0.55 mm.
- Depth of tank = 2.5 m to 3.5 m.
- Area = 10 to 80 m^2 each unit.
- Rate of filtration 3000 to 6000 lit/m^2/hour (slow sand filter $\times$ 30)

• Cross-sectional area of Manford = 2 $\times$ cross-sectional area of lateral.

• Cross-sectional area of each lateral = 2 to 4 times cross-sectional area of perforations in it.

• Total cross-sectional area of perforation = 0.2% of the total area of 1 filter bed

• $$\frac{Length\ of\ each\ lateral}{Dia\ of\ lateral} > 60$$

• 4.5% of filtered water is used as a backwash.

• 30 min. used for backwash.

Hydraulics of Sand Gravity Filters

$$\boxed{h_L = \frac{1.067V^2}{\phi \cdot g \cdot n^4} \in \frac{C_D \cdot f}{d}}$$

Where,

h_L = Frictional head loss through the filter in the meter.

V = Approach velocity or filtration velocity in m/s.

Depth of filter in meter

ϕ = Shape factor (for non-spherical particle)

d = The diameter of sand particles in the meter.

g = Accelerations due to gravity in m/s2.

n = Porosity

C_D = Newton's dray coefficient.

f = Mass friction of sand particle of dia d.

Rose Equation,

$$\boxed{h_L = \left|\frac{1.067V^2D}{\phi g n^4} \cdot \frac{C_D}{d}\right]}$$

Hydraulic head loss and expansion of the filter during backwash of RSF

- $\boxed{H_{L_e}\gamma_w = D\gamma_{sub}}$

Where,

H_{L_e} = Head loss through the filter bed required to initiate expansion in the meter.

γ_w = Unit weight of meter in kN/m^3.

D = Depth of filter bed in the meter.

γ_{sub} = Submerged unit weight of sand in bed of depth 'D'

- $\boxed{H_{L_e} = D(1-n)(G-1)}$
- $\boxed{H_{L_e} = D_e(1-n_e)(G-1)}$

Where,

D_e =

Depth of expanded/fluidized bed in the meter.

The porosity of the expanded fluidized bed.

$$\boxed{D_e = \frac{(1-n)D}{(1-n_e)}}$$

$$\boxed{D_e = (1-n)D \cdot \sum \frac{f}{1-n_e}}$$

Where,

f = mass fraction of sand of various sizes in the sand (as per sieve analysis0

- $\boxed{n_e = \left(\frac{V_b}{V_s}\right)^{0.22}}$
- $\boxed{V_s = \left[\frac{4}{3}\frac{gd(G-1)}{C_D}\right]^{1/2}}$

where,

n_e = The porosity of the expanded bed

V_b =

Backwash velocity in m/s

V_s = Setting velocity in m/s.

Disinfection or Sterilization

(i) Treatment with Ozone

$$3O_2 \xrightarrow[\text{Arc voltage}]{\text{Under high electric}} 2O_3$$

$$\begin{array}{ll} \text{Oxygen} & \text{Ozone} \\ \text{Molecule} & \text{Molecule} \\ 3\,\text{Mole} & 2\ \text{Mole} \end{array}$$

$$\begin{array}{ccccc} O_3 & \longrightarrow & O_2 & + & [O] \\ \text{Ozone} & & \text{Oxygen} & & \text{Nascent oxygen} \end{array}$$

(ii) Disinfecting Action of Chlorine

$$Cl_2 + H_2O \xrightarrow{PH > 5} HOCI + HCI$$

$$\text{Hypochlorous acid.}$$

$$HOCI \underset{PH<7}{\overset{PH>8}{\rightleftharpoons}} H^+ \quad + \quad OCI^-$$

$$\text{Hydroge ion} \quad \text{Hypochlorite ion}$$

$$NH_3 + HOCI \rightarrow NH_2CI + H_2O \quad PH > 7.5$$

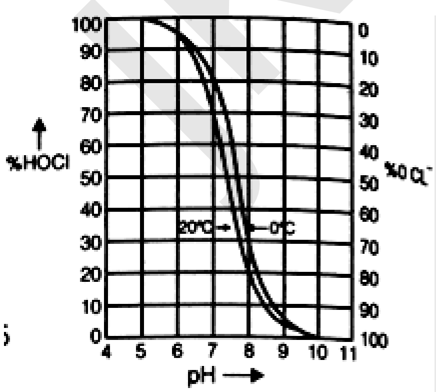
$$\text{Monochloro Amine}$$

$$NH_2CI + HOCI \rightarrow NHCI_2 + H_2O \ PH \rightarrow 5 \text{ to } 6.5$$

$$Di - chloroamine$$

$$NHCI_2 + HOCI \rightarrow NCI_3 + H_2O \quad PH < 4.4$$

$$\text{Nitrogen Trichloroamine}$$



(iii) Doses of Chlorine

Type of virus to be killed	Quantity of free chlorine required in mg/l with about 30 minutes contact period for water of pH lower than 7 or so
Poliomyelitis virus	0.1
Hepatitis virus	0.4
Cysts of E.histolytica, i.e. the organism causing ameobic dysentery	3.0 or even lower
Tuberculosis organisms	3.0
Coxsaickie Virus	Very huge dose varying from 21 to 138 mg/l.

(iv) Forms in which chlorine is applied

(a) Free chlorine

(b) Hypochlorites & Bleaching Powder

(c) Chloramines

(d) Chlorine dioxide

$$\underset{\text{Calcium Hypochlorite}}{Ca(OCl)_2} \xrightleftharpoons{H_2O} \underset{\text{Calcium ion}}{Ca^{2+}} + \underset{\text{hypochlorite ion}}{2OCl^-}$$

$$OCl^- + H^+ \xrightleftharpoons[PH > 8]{PH < 7} HOCl \text{ (Hypochlorous acid)}$$

Type of Chlorination

(i) Plain chlorination

(ii) Pre-chlorination

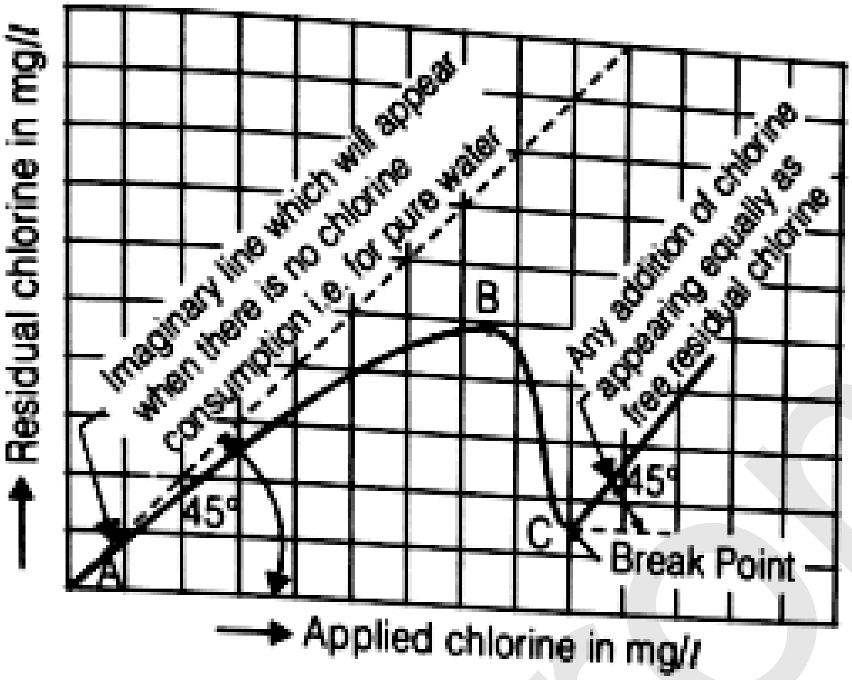
(iii) Post-chlorination

(iv) Double chlorination

(v) Break point chlorination

(vi) Super chlorination

(vii) Dechlorination



Test of Chlorine Residual

1. Orthotoulidine test: color – matching method
2. Arsenide ortho-toulodine test – when mineral present in the water sample, Also a color matching method.
3. DPD and chlorotic test (Di-ethyl phenylene diamine): color making method.
4. Starch iodide Test

$$\left[\begin{array}{l}\textit{Quantity of chlorine}\\ \textit{in mg / lit in the}\\ \textit{original sample of}\\ \textit{water}\end{array}\right] = 0.355\left[\begin{array}{l}\textit{Number of ml of}\\ \textit{thiosulphate req}\\ \textit{uired to remove}\\ \textit{the blue colour}\end{array}\right]$$

Water Softening

Methods of Removing Temporary hardness

(i) Boiling

$Ca(HCO_3)_2 + Heat \rightarrow CaCO_3 \downarrow CO_2 \uparrow + H_2O$

Calcium bi Carbonate   *Calcium Carbonate*

(ii) Addition of lime

$MgCO_3 + Ca(OH)_2 \rightarrow Mg(OH_2) \downarrow + CaCO_3 \downarrow$

Magnesium Hydrate   Magnesium Calcium

$Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow Ca(HCO_3)_2 + Mg(OH)_2 \downarrow$

$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow 2CaCO_3 \downarrow + 2H_2O$

Method of Removing Permanent Hardness

(i) Lime-Soda Process

(a) $Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow 2CaCO_3 \downarrow + 2H_2O$

(b) (i) $Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow Ca(HCO_3)_2 + Mg(OH)_2 \downarrow$

(ii) $MgCO_3 + Ca(OH)_2 \rightarrow Mg(OH)_2 + CaCO_3 \downarrow$

(c) $MgCI_2 + Ca(OH)_2 \rightarrow Mg(OH)_2 \downarrow + CaCI_2$

(d) $MgSO_4 + Ca(OH)_2 \rightarrow Mg(OH)_2 \downarrow + CaSO_4$

(e) $CO_2 + Ca(OH)_2 \rightarrow CaCO_3 \downarrow + H_2O$

(f) $CaCI_2 + Na_2CO_3 \rightarrow CaCO_3 \downarrow 2NaCI$

(g) $CaSO_4 + Na_2CO_3 \rightarrow CaCO_3 \downarrow + Na_2SO_4$

$$\boxed{\text{Dry sludge produced in mg / lit} = \left[C_{aR} + 0.58M_{gR} + L_{iA}\right]}$$

where,

C_{aR} = calcium hardness removed in mg/lit (expressed as

M_{gR} =

Magnesium hardness removed in mg/lit (expressed as v$CaCO_3$)

L_{iA} = Lime added in mg/lit (expressed as $CaCO_3$)

Zeolite or Base Exchange or Cation-Exchange Process for Removing Hardness

$$\underset{\text{Sodium zeoliteor active zeolite}}{Na_2z} + \underset{\text{Calcium or Magnesium Salt}}{\begin{matrix} Ca \\ Mg \end{matrix}\begin{cases} (HCO_3)_2 \\ SO_4 \\ CI_2 \end{cases}} \rightarrow \underset{\text{Sodium slats which don't Cause hardness}}{Na_2\begin{cases} (HCO_3)_2 \\ SO_4 \\ CI_2 \end{cases}} + \underset{\text{Calcium or Magnesium zeolite}}{\begin{matrix} Ca \\ Mg \end{matrix}\Big\{ z}$$

$$\underset{\text{Used zeolite}}{\left.\begin{matrix} Ca \\ Mg \end{matrix}\right\} z} + \underset{\text{Sodium chloride solution}}{2NaCI} \longrightarrow \underset{\text{Regenerated zeolite}}{Na_2z} + \left.\begin{matrix} Ca \\ Mg \end{matrix}\right\} CI_2$$

Demineralization Process for Removing Hardness

$$Ca(HCO_3)_2 + \underset{\text{Fresh cation exchange resin}}{H_2R} \longrightarrow \underset{\text{Exhayseted rasin}}{CaR} + 2H_2O + 2CO_2 \uparrow$$

$$CaCI_2 + H_2R \longrightarrow CaR + 2HCI$$

$$MgSO_4 + H_2R \longrightarrow CaR + H_2SO_4$$

$$2NaCI + H_2R \longrightarrow Na_2R + 2HCI$$

$$\underset{\text{Fresh anion Exchange resin}}{R - OH} + HCI \longrightarrow \underset{\text{Exhausted resin}}{RCI} + \underset{\text{water}}{H_2O}$$

$$\underset{\text{Fresh anion Exchange resin}}{2R - OH} + H_2SO_4 \longrightarrow \underset{\text{Exhausted resin}}{R_2SO_4} + \underset{\text{water}}{2H_2O}$$

Drinking water specification: IS: 10500, 1992 (Reaffirmed 1993)

Tolerance Limit

S. No.	Parameter	IS:10500 Requirement (desirable limit)	Undesirable Effect outside the desirable limit	IS:10500 Permissible limit in the absence of alternate source
Essential Characteristics				
1.	pH	6.5-8.5	Beyond this range the water will effect the mucous membrance and/or water supply system	No relaxtion
2.	Colour (hazen units), Maximum	6.5-8.5	Above 5, consumer acceptance decreases	25
3.	Odour	Unobjectionable	----	---
4.	Taste	Agreeable	----	---
5.	Turbidity, NTU, Max	5	Above 5, consumer acceptance decreases	10

Following Results are expressed in mg/l:				
6.	Total hardness as $CaCO_3$ Max	300	Encrustation in water supply structure and adverse effects on domestic use	600
7.	Iron as Fe, Max	0.30	Beyond this limit taste/appearance are affected, has adverse effect on domestic uses and water supply structures, and promotes iron bacteria.	1.0
8.	Chlorides as Cl, Max	250	Beyond this limit tast, corrosion and palatability are effected	1000
9.	Residual, Free Chlorine, Min	0.20	---	---

Desirable Characteristics				
10.	Dissolved solids, Max	500	Beyond this palatability decreases and may cause gastro intentiona irritation	2000
11.	Calcium as Ca, Max	75	Encurustation in water supply structure and adverse effects on domestic use	200
12.	Magnesium as Mg, Max	30	---	100
13.	Nitrates as NO_3	45	Beyond this methanemoglobinemia takes place	100
14.	Fluoride, Max	1.0	Fluoride may be kept as low as possible. High fluoride may cause fluorosis	1.5
15.	Alkalinity, Max	200	Beyond this limit taste becomes unpleasant.	600

wastewater characteristics

Aerobic Decomposition

(i) Nitrogenous organic matter
$$\xrightarrow[\text{Aerobic}]{\text{Oxidation by}} NO_3^- + NH_3 \uparrow + Energy$$

Nitrate

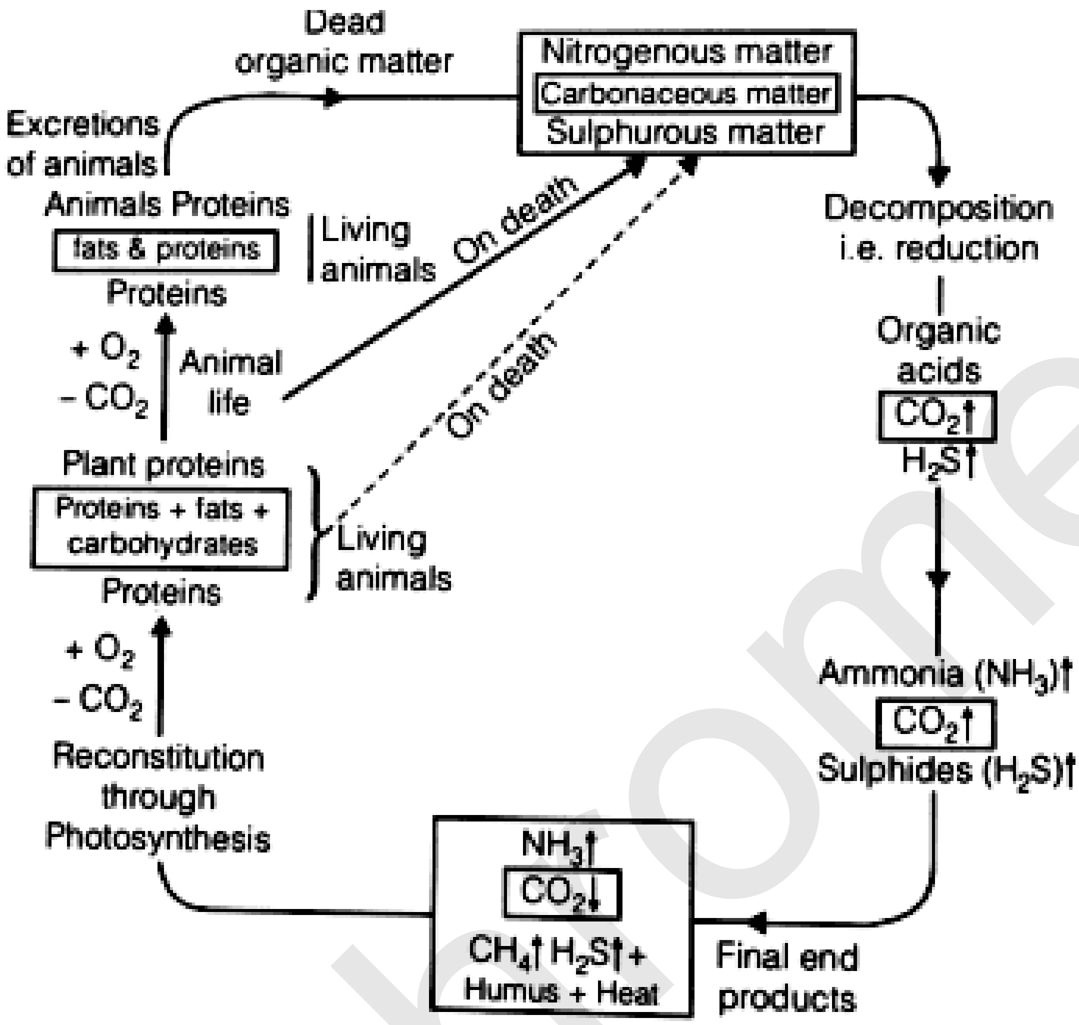
(ii) Carbonaceous organic matter
$$\xrightarrow[\text{Aerobic}]{\text{Oxidation by}} CO_2 + H_2O + Energy$$

Carbon dioxide

(iii) Sulphurous organic matter
$$\xrightarrow[\text{Aerobic}]{\text{Oxidation by}} SO_4^{2-} + Energy$$

Sulphate

Nitrogen Cycle under Aerobic Decomposition



Anaerobic Decomposition

(i) Nitrogenous organic matter

$$\xrightarrow[\text{Anaerobic}]{\text{Reduction by}} N_2 \uparrow + NH_3 + Organic\ acids$$
$$+\ Heat\ energy$$

(ii) Carbonaceous organic matter

$$\xrightarrow[\text{Anaerobic}]{\text{Reduction by}} CO_2 \uparrow +\ Heat\ energy$$

(iii) Sulphurous organic matter

$$\xrightarrow[\text{Anaerobic}]{\text{Reduction by}} H_2S \uparrow +\ Heat\ energy$$

(iv) Organic acids

$$\xrightarrow[\text{Anaerobic bacteria}]{\text{Methane for min g}} CH_4 \uparrow + CO_2 \uparrow + Heat\, energy$$

Threshold Odour Number (TON)

$$\boxed{TON = \frac{V_S + V_D}{V_S}}$$

V_S = The volume of the sewage

V_D = The volume of distilled water or odourless water.

Total Solids, Suspended Solids and Settleable Solids

(i)

$$\boxed{S_3 = S_1 - S_2}$$

Where,

S_3 = Dissolved solids plus colloidal or filterable solids in mg/lit

S_2 = Non-filterable solids in mg/lit

S_1 = The total amount of solids in mg/lit

S_4 = Volatile suspended solids, in (mg/lit.)

S_5 = Fixed solids

(ii) $\boxed{S_2 - S_4 = S_5}$

Type of solid	Strength of Sewage		
	Weak	Weak	Strong
Total solids	400	800	1200
Suspended solids			
Total	100	200	350
Volatile	75	135	210
Settleable solids	2.5	5	7
Ether soluble matter such as fats, oils and grease.	6	14	20

Total Solids

50% → Dissolved

25% → Suspended

25% → Settlable

Chemical Oxygen Demand

(i) Biodegradable + non Biodegrable O.M.

(ii) $K_2Cr_2O_7 + H_2SO_4$ added and used is measured.

Theoretical Oxygen Demand

$$C_xH_y + \left(x + \frac{y}{4}\right)O_2 \longrightarrow xCO_2 + \frac{y}{2}H_2O$$

$$C_6H_6 + 7.5O_2 \longrightarrow 6CO_2 + 3H_2O$$

Benzene

$$C_6H_{12}O_6 + 6O_2 \longrightarrow 6CO_2 + 6H_2O$$

Glucose

Biochemical Oxygen Demand

- $\boxed{BOD = (DO_i - DO_f)\,Dilution\ Factor}$

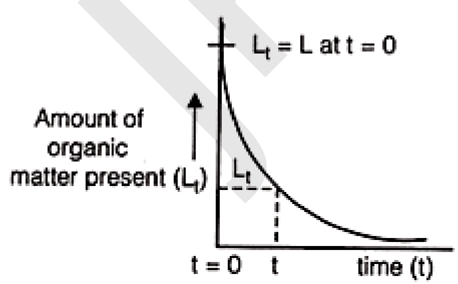
Where,

Biochemical oxygen demand in ppm or mg/lit.

Initial dissolved oxygen in mg/lit.

Final dissolved oxygen in mg/lit.

- $\boxed{Dilution\ Factor = \dfrac{Volume\ of\ the\ diluted\ sample}{Volume\ of\ the\ undiluted\ sewege\ sample}}$



Ist Stage BOD Curve

(i) $\boxed{\frac{dL_t}{d_t} = -kL_t}$

Where,

k = Rate constant signifying the rate of oxidation of organic matter and it depends upon the nature of organic matter and temperature. Its unit is per day.

$L_t = O_2$ the equivalent of organic matter present after t days.

(ii)

$$\boxed{L_t = L \cdot (10)^{-k_D t}}$$

Where,

$k_D =$ Deoxygenation constant.

$L =$ Organic matter present at

$t = 0$

(iii)

$$\boxed{k_D = 0.434K}$$

(iv)

$$\boxed{Y_t = L - L_t}$$

Where,

$Y_t =$

The total amount of organic matter oxidized int days i.e. BOD.

(v)

$$\boxed{Y_t = L\left[1 - (10)^{-k_D t}\right]}$$

(vi)

$$\boxed{Y_u = L}$$

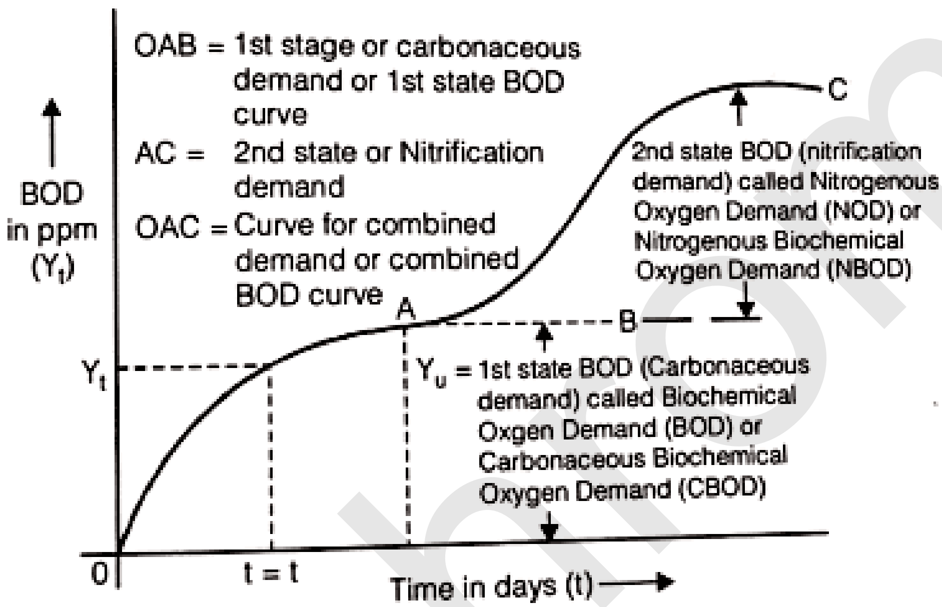
Where,

$Y_u =$ Ultimate B.O.D of $t = \infty$ days.

(vii)

$$\boxed{k_{D(T^\circ C)} = k_{D(20^\circ C)}\left[1.047\right]^{T-20^\circ C}}$$

Water type	K_D value per day
Tap waters	<0.05
Surface waters	0.05 – 0.1
Municipal waste waters	0.1 – 0.15
Treated sewage effluents	0.05 – 0.1

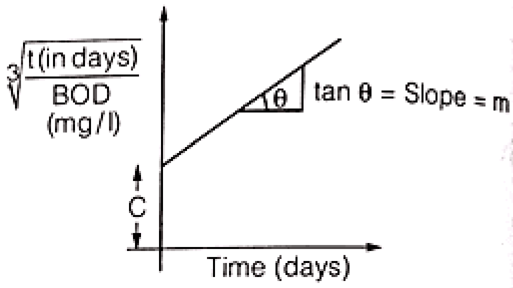


Parameter/Characteristic	Strength of Sewage		
	Weak	Medium	Strong
Total suspended solids (SS)	100	200	350
Volaite suspended solids	75	135	210
BOD	100	200	400
COD	175	300	600
TOC*	100	200	400
Ammonia-N	5	10	20
Organic-N	8	20	40
PO_4-P	7	10	20

(viii) Laboratory Estimations of k_D and L values

(Thomas Method)

$$\boxed{k_D = 2.61\frac{m}{C}}$$



Where,

m = Slope of the line

C = Intercept of the line on the y-axis.

$$\boxed{L = Y_u = \frac{1}{2.3k_D C^3}}$$

Relative Stability (s)

$$\boxed{S = 100\left[1-(0.794)^{t_{20}}\right]}$$

$$\boxed{S = 100\left[1-(0.630)^{t_{37}}\right]}$$

Where time in days at 20°C.

t_{37} = time in days at 37°C.

disposal of sewage effluents and design of sewer system and appurtenances

Disposal of The Sewage Effluents

Standards of Dilution for Discharge of Wastewaters into Rivers

- Standards of Dilution based on Royal Commission Report

Dilution factor	Standards of purification required
Above 500	No treatment is required. Raw sewage can be directly discharged into volume of dilution water.
Between 300 to 500	Primary treatment such as plain sedimentation should be given to sewage, and the effluents should not contain suspended solids more than 150 ppm.
Between 150 to 300	Treatments such as sedimentation, screening and essentially chemical precipitation are required. The sewage effluent should not contain suspended solids more than 60 ppm.
Less than 150	Complete through treatment should be given to sewage. The sewage effluent should not contain suspended solids more than 30 ppm., and its 5 days BOD at 18.3°C should not exceed 20 ppm.

- **BIS Standards** for Discharge of Sewage and Industrial Effluents in Surface Water Sources and Pub

Characteristic of the Effluent	Tolerance limit for sewage effluents discharged into surface water sources, as per IS 4764-1973	Tolerance limit for Industrial effluents discharged into	
		Inland surface waters, as per IS 2490-1974	Public sewers as per IS 3306-1974
(1)	(2)	(3)	(4)
BOD_5	20 mg/l	30 mg/l	500 mg/l
COD	-	250 mg/l	-
pH value	-	5.5 to 9.0	5.5 to 9.0
Total suspended solids (TSS)	30 mg/l	100 mg/l	600 mg/l
Temperature	-	40°C	45°C
Oil and grease	-	10 mg/l	100 mg/l
Phenolic compounds (as Phenol)	-	1 mg/l	2 mg/l
Cyanides (as CN)	-	0.2 mg/l	-
Sulphides (as S)	-	2 mg/l	-
Fluorides (as F)	-	2 mg/l	-
Total residual chlorine	-	1 mg/l	-
Insecticides	-	Zero	-

- **General standards for Discharge of Environment Pollutants** from effluents into Surface Water Sources, Public Sewers, and Marine Coasts Under Environment (Protection) Rules, 1986

Characteristic of the Effluent i.e. Name of pollutant in the effluent	Standard Prescribed under Environment (Protection Rules, 1986 of GOI for		
	Inland surface waters	Public sewers	Marine coasts i.e. seas and oceans
(1)	(2)	(3)	(4)
Colour and odour	All efforts should be made to remove colours and unpleasant odours, as for as possible.	All efforts should be made to remove colours and unpleasant odours, as for as possible.	All efforts should be made to remove colours and unpleasant odours, as for as possible.
Total suspended solids (TSS)	100 mg/l	600 mg/l	(i) 100 mg/l for process waste water. (ii) For cooling water effluent, 10% above total suspended mater of influent.
Particle size of suspended solids	Shall pass 850 micron sieve	-	(a) Floatable solids: max 3mm (b) Settleable solids: max 850 micron.
BOD5 at 20°C	30 mg/l	350 mg/l	100 mg/l
COD	250 mg/l		250 mg/l
pH value	5.5 – 9.00	5.5 – 9.00	5.5 – 9.00
Temperature	Shall not exceed 5°C above the temperature of receiving water.	-	Shall not exceed 5°C above the temperature of receiving water.
Oil and grease	10 mg/l	20 mg/l	20 mg/l
Total residual chlorine	1.0 mg/l	-	1.0 mg/l
Ammonical Nitrogen (as N)	50 mg/l	50 mg/l	50 mg/l
Total Kjeldahl nitrogen (as N)	100 mg/l	-	100 mg/l
Free Ammonia (as NH_3)	5.0 mg/l	-	5.0 mg/l
Phenolic compounds (as C_6H_5OH)	1.0 mg/l	5.0 mg/l	5.0 mg/l
Sulphide (as S)	2.0 mg/l	-	5.0 mg/l

Dilution and Dispersion

$$\boxed{C_{mix} = \frac{C_S \cdot Q_S + C_R Q_R}{Q_S + Q_R}}$$

Where,

C_S = The concentration of sewage in mg/lit.

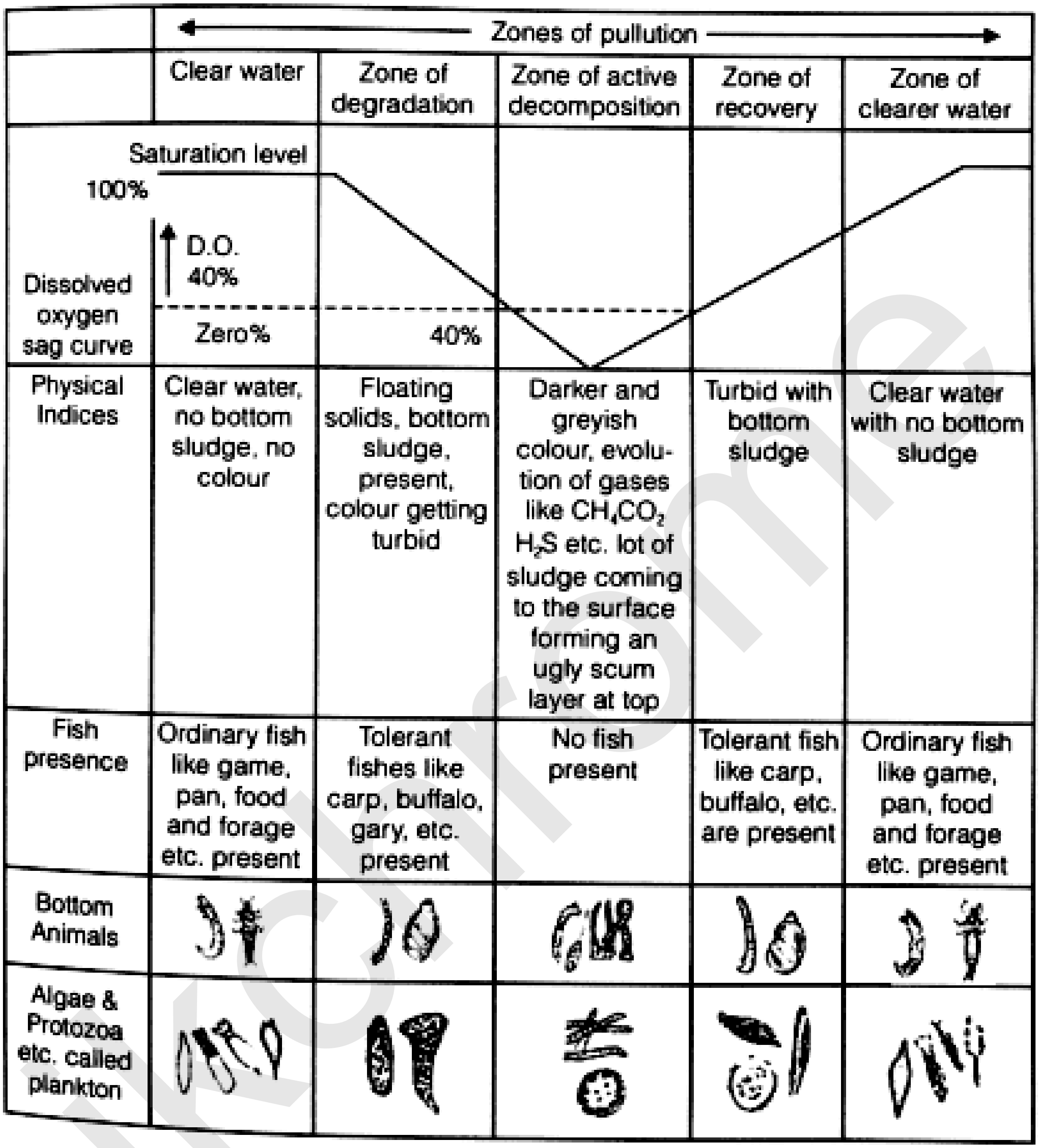
Q_S = A flow rate of sewage in m^3/sec or lit/sec.

C_R = The concentration of the river in mg/lit.

Q_R = Flow rate (discharge in m^3/sec or lit/sec.

C_{mix} = The concentration of the mixture.

Zone of Pollution in River Stream

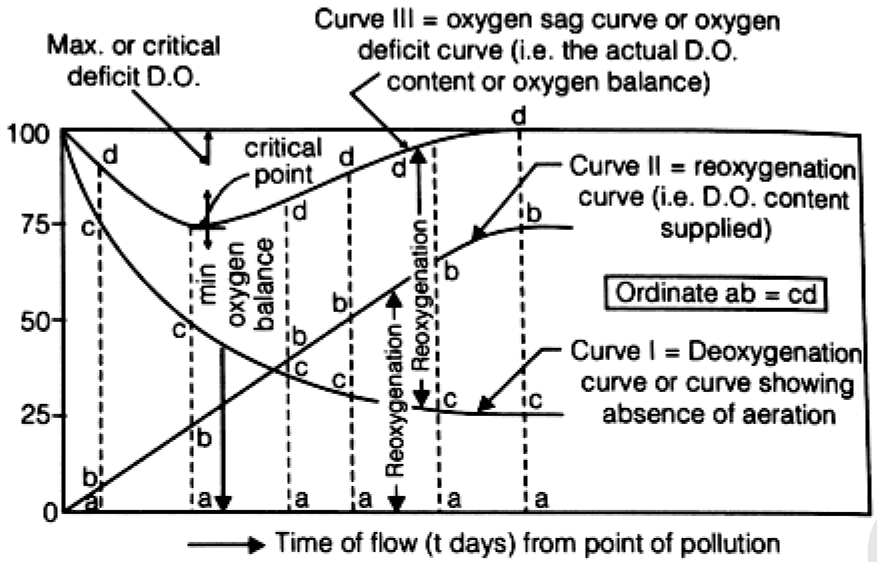


$$\boxed{\text{Oxygen deficit} = \text{Saturation D.O} - \text{Actual D.O}}$$

Saturation D.O at 20°C → 9.2 mg/lit.

Saturation D.O at 30°C → 7.6 mg/lit.

Saturation D.O at 0°C → 14.6 mg/lit.



$ab = cd$

- $$\boxed{TOD > COD > BOD}$$

- $$\boxed{COD > (BOD)_U > BOD_5}$$

Where,

$TOD =$ Theoretical oxygen demand

$BOD =$ Biological oxygen demand

$COD =$ Chemical oxygen demand

$(BOD)_U =$ Ultimate BOD (Y_U).

- $$\boxed{\begin{bmatrix} Standard\ BOD \\ (5\ days) \\ of\ industrial \\ sewage \end{bmatrix} = \begin{bmatrix} Standard\ BOD \\ (5\ days)\ of \\ domestic\ sewage \\ per\ person\ per\ day \end{bmatrix} \times Population\ equivalent}$$

Stretcher-PHELPS EQUATION

$$\boxed{D_t = \frac{k_D L}{k_R - k_D}\left[(10)^{-k_D t} - (10)^{-k_R t}\right] + \left[D_o \cdot (10)^{-k_R t}\right]}$$

- $$\boxed{k_{D(T^\circ C)} = k_{D(20^\circ C)}\left[1.047\right]^{(T-20^\circ C)}}$$

- $$\boxed{k_{R(T^\circ C)} = k_{R(20^\circ C)}\left[1.016\right]^{(T-20^\circ C)}}$$

- $$\boxed{f = \frac{k_R}{k_D}}$$

- $$\boxed{t_C = \frac{1}{k_D(f-1)}\log_{10}\left[\left\{1-(f-1)\frac{D_O}{L}\right\}f\right]}$$

- $$\boxed{D_C = \frac{L}{f}\cdot\left[10\right]^{-k_D \cdot t_C}}$$

Where,

D_t = D.O deficit in mg/lit after t days.

L = Ultimate first stage BOD of the mix at a point of waste discharge in mg/lit.

D_O = Initial oxygen deficit of the mix at the mixing point in mg/lit.

k_R = Reoxygenation constant

k_D = Deoxygenation constant

f = Self-purification constant

t_C =

Critical time at which minimum dissolved oxygen occurs i.e.

$$\boxed{\frac{dD_t}{dt} = 0}$$

$D_c =$

Critical maximum oxygen deficit.

Constituent Pollutant contained in the Wastewater Effluent	Tolerance limit
(2)	(3)
BOD_5	100 mg/l
COD	250 mg/l
pH value	5.5 to 9.0
Total suspended solids	100 mg/l
Oil and grease	20 mg/l
Fluorides (as F)	15 mg/l
Ammoniacal Nitrogen (as N)	50 mg/l

Type of soil	Doses of sewage in cubic metres per hectare per day	
	Raw sewage	Settled sewage
Sandy	120 – 150	220 – 280
Sandy loam	90 – 100	170 – 220
Loam	60 – 80	110 – 170
Clayey loam	40 – 50	60 – 110
clayey	30 – 45	30 – 60

Characteristics of effluent i.e. pollutant of waste water	Standards under Environment (Protection) Rules, 1986
(2)	(3)
Colour and odour	All efforts should be made to remove colour and unpleasant odour as far as practicable
BOD_5 at 20°C	100 mg/l
Suspended Solids (SS)	200 mg/l
pH value	5.5 to 9.0
Oil and grease	10 mg/l
Arsenic (As)	0.2 mg/l
Cyanide (as CN)	0.2 mg/l
Radioactive materials	
(a) Alpha emitters	10^{-8} μC/m/
(b) Beta emitterres	10^{-7} μC/m/

DESIGN OF SEWERAGE SYSTEM AND APPURTENANCES

1.INTRODUCTION

The sewer pipes are laid below the ground level sloping continuously at sufficiently steeper gradient. It is different from water supply conduit as sewage pipes are designed to flow under gravity only. Also, sewage contains lots of suspended particles which may settle down and clog the system. To avoid the clogging, sufficient velocity known as **'Self cleansing velocity'** is need to be maintained in the system.

1. **HYDRAULIC DESIGN OF SEWERS**

2.1. Important Formulas for Determining Flow Velocity

Following formulas used to determine flow velocities in sewers:

(i) Manning's formula: The flow velocity is given by

Where,

R = Hydraulic radius = A/P

A = Cross sectional area of sewer

P = Wetted Perimeter

S = Ground slope

n = manning's constant

(ii) Chezy's formula: The flow velocity is given by

Where,

C = Chezy's constant

2.2. Design Data

Sewage should be designed for maximum hourly discharge and it should be ensured that velocity of flow will always be greater than self-cleansing velocity.

Maximum hourly discharge = 3 × Average daily discharge

Maximum daily discharge = 2 × Average daily discharge

It is assumed that 80% of water supply goes to sewers

The self-cleansing velocity can be calculated using the Shield's formula

Where,

G = Specific gravity of particle

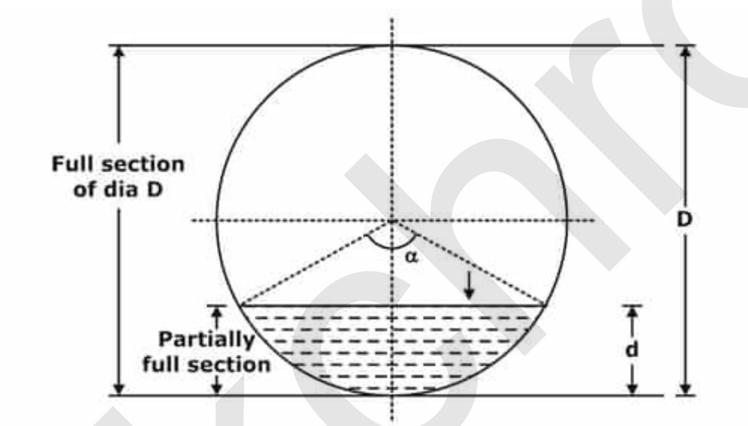
d_p = Size of particle

K = A constant

R = Hydraulic radius of sewer

n = manning coefficient

2.3. Circular Sewer running Partially Full



When the sewage is running partially full at depth d such that,

Proportional depth

Area of flow

Proportional area

Wetted Perimeter

Proportional wetted perimeter

Hydraulic radius

Proportional hydraulic radius

Proportional velocity of flow

(Using manning's formula)

Proportional discharge

Note:

(i) For constant value of manning's coefficient, the velocity will be maximum when d = 0.81D.

(ii) For constant value of manning's coefficient, the discharge will be maximum when d = 0.95D.

Equal Degree of Self Cleansing:

For equal degree of self Cleansing, the drag force under partial flow should be same as drag force under full flow.

The proportional velocity in the above case will be equal to

If, the slope of both the sewer is same,

r = R

This is possible only when the sewer is running either half full or completely full.

Treatment of Sewage

Sedimentation Tank

Settling Velocity

(i)

$$\boxed{V_s = \frac{g}{18}(G-1)\frac{d^2}{\nu}}$$

for d < 0.1 mm

Where,

$V_s =$

The velocity of the settlement of particle or settling velocity in m/sec.

$d =$ The diameter of the particle in the meter.

$G =$ Specific gravity of the particle.

$v =$ Kinematic viscosity of water in m^2/sec.

$$\boxed{v = \frac{Y}{\delta}}$$

$Y =$

Dynamic viscosity

$\delta =$

Density

(ii)

$$\boxed{V_s = \sqrt{\frac{\frac{4}{3}g \cdot (G-1)d}{C_D}}}$$

where,

$$\boxed{\begin{aligned} C_D &= \textit{Coefficient of drag} \\ &= \frac{24}{R_e} \end{aligned}}$$

→ For laminar flow

$R_e =$ Reynolds number

$$\frac{\delta v_s d}{Y} = \frac{V_s d}{v}$$

$$\boxed{C_D = \frac{24}{R_e} + \frac{3}{(R_e)^{0.5}} + 0.34} \boxed{C_D = \frac{18.5}{R_e^{0.6}}} \longrightarrow$$

for transition flow.

$$\boxed{C_D = 0.34 + 0.4} \longrightarrow$$

for turbulent flow.

(iii)

$$\boxed{V_s = \left[\frac{g(G-1)}{13.88}\frac{d^{1.6}}{\nu^{0.6}}\right]^{0.714}} \; 0.1\ mm < d < 1.0\ mm.$$

(iv) Newtons Equation for Turbulent Settling

$$\boxed{V_s = 1.8\sqrt{gd(G-1)}} \;\; for\ \ d > 1\ mm$$

(v) Modified Hazen's Equation for Transition Zone

(a)

$$\boxed{V_s = 60.6d(G-1)\cdot\left(\frac{3T+70}{100}\right)}$$

$$For\ 0.1 < d < 1\,mm$$

Where T = Temperature in °C.

(b) Putting G = 2.65 for Inorganic Solids

$$\boxed{V_s = d(3T+70)}$$

(c) Putting G = 1.2 for Organic Solids

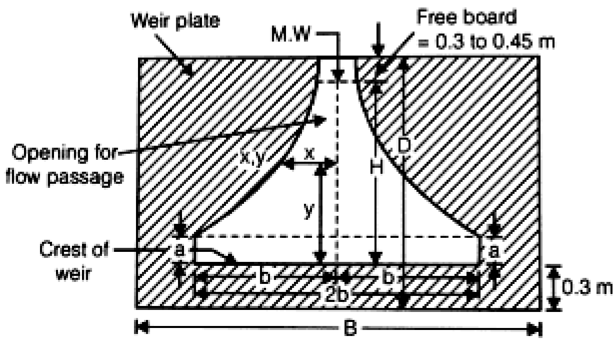
$$\boxed{V_s = 0.12d(3T+70)}$$

Critical Scour Velocity in Constant Velocity Horizontal Flow Grit Chamber

(V_H)

$$\boxed{V_H = 3\ to\ 4.5\sqrt{gd(G-1)}}$$

PROPORTIONAL FLOW WEIR



$$\boxed{x = \frac{2BV_h}{C_d \cdot \sqrt{2g} \cdot \pi\sqrt{y}}}$$

$$\boxed{b = 1.467\ B\ V_h}$$

Where,

$B =$ Width of the channel.

$V_h =$ Horizontal flow velocity.

$C_d =$ Coefficient of discharge.

x and y are coordinates on weir profile.

Parabolically or V-Shaped Grit Chamber Provided with a Parshall Flume

(i) Parshall Flume

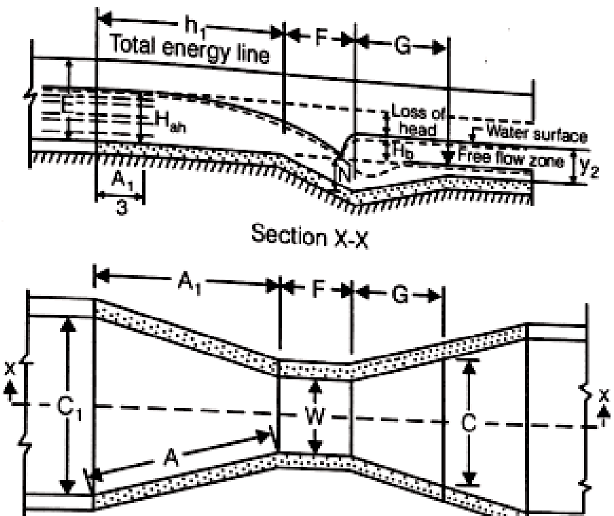
$$\boxed{Q = 2.264\,W\left(H_a\right)^{3/2}}$$

Where,

$W =$ Width of the throat in the meter.

Flow in (m^3/sec) through Parshall flume.

$H_a =$ Depth of flow in the upstream leg of a flume of one-third portion in the meter.



The Parshall flume

(ii) Parabolic Grit Channel

$$\boxed{Q = C_1 \cdot \int_{y=0}^{y=y} x \, dy} \text{ where } \boxed{V_h = \frac{Q}{\int_0^y x \; dy} = C_1}$$

$$\boxed{Q = C_2 y^n}$$

$$\boxed{Y^{n-1} = \frac{C_1}{C_2} \cdot X}$$

Where,

n = Discharge coefficients of the control section.

= 1.5 for partial flume.

= 1 for proportional flow weir.

- Aerated Grit Channels

$$\boxed{Detention\ period = 3 \min utes}$$

- Detritus Tank

$$\boxed{Detention\ period = 3\ to\ 4 \min utes}$$

Skimming Tank

(i) Detention Period = 3 to 5 minutes.

(ii) Amount of compressed air required = 300 to 6000 m^3 per million liters of sewage.

(iii) Surface Area,

$$\boxed{A = 0.00622 \frac{q}{v_r}}$$

Where,

$q =$ Rate of flow of sewage in m^3/day.

$V_r =$ Min. rising velocity of greasy material to be removed in m/min

= 0.25 m/min mostly.

Vacuators

Vacuum Pressure = 0 to 25 cm of Hg

For 10 to 15 minutes.

- **Sedimentation Tank**

(i) Overflow rate

= 40000 to 50000 lit/m^2 day for plain sedimentation.

= 50000 to 60000 lit/m^2 day for sedimentation with coagulation.

= 25000 to 35000 lit/m^2 day for secondary sedimentation tank

(ii) Depth ~ 2.4 to 3.6 m.

(iii) Detention time = 1 to 2 hour.

(iv) width = 6.0 m

(v) Length = 4 to 5 times width.

(vi) Velocity of flow Vf = 0.3 m/min.

(vii) $\boxed{V = \frac{Q}{BH}}$

Where,

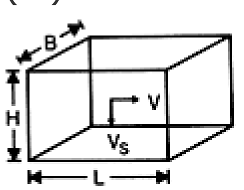
V = Flow velocity

B = Width of the Basin

H = Depth of sewage in the tank.

(viii) $\boxed{\dfrac{V}{V_s} = \dfrac{L}{H}}$

(ix) $\boxed{V_s = \dfrac{Q}{BL}}$



- **Detention Time**

(a) $\boxed{t = \dfrac{BLH}{Q}}$

For rectangular Tank

(b) $\boxed{t = \dfrac{d^2\left(0.011d + 0.785H\right)}{Q}}$ for circular tank

Where

d = Dia of the tank

H = Vertical depth of wall or side depth

- **Displacement Efficiency** (η)

$$\boxed{\eta = \frac{\textit{Flowing through period}}{\textit{Detention period}}}$$

Trickling Filter

(a) Conventional Trickling Filter or Low Rate Trickling Filter

$$\boxed{\eta(\%) = \frac{100}{1 + 0.0044\sqrt{u}}}$$

Where,

$\eta =$ The efficiency of the filter and its secondary clarifier, in terms of % of applied BOD

$u =$ Organic loading in kg/ha-m/day applied to the filter (called unit organic loading)

(b) High Rate Trickling Filter

(i) $\boxed{F = \dfrac{\left(1+\dfrac{R}{I}\right)}{\left(1+0.1\dfrac{R}{I}\right)^2}}$

Where, F = Recirculation factor

Recirculation ratio

(ii) $\boxed{\eta(\%) = \dfrac{100}{1+0.0044\sqrt{\dfrac{Y}{VF}}}}$

Where,

Y = Total organic loading in kg/day applied to the filter i.e. the total BOD in kg.

$\dfrac{Y}{VF}$ = Unit organic loading in kg/Ha-m/day

V = Filter volume in Ha-m.

% efficiency of single-stage high rate trickling filter.

(iii)

$$\boxed{\eta'(\%) = \dfrac{100}{1+\dfrac{0.0044}{1-\eta}\sqrt{\dfrac{Y'}{V'F'}}}}$$

Where,

η' = Final efficiency in the two-stage filter.

Y' = Total BOD in the effluent from the first stage in kg/day.

F' = Recirculation factor for second stage filter

V' = Volume in second stage filter in ha-m.

Characterristics (2)	Conventional or Standard filters (3)	High filters (4)
Depth of filter media	Varies between 1.6 to 2.4 m.	Varies between 1.2 to 1.8 m
Size of filter media	25 to 75 mm.	25 to 60 mm.
Land required	More land area is required, as the filter loading is less.	Less land area is required as the filter loading is more.
Cost of operation	It is more for treating equal quantity of sewage.	It is less for treating equal quantity of sewage.
Method of operation	Continuous application, less flexible requiring less skilled supervision.	Continuous application, more flexible, and more skillful operation is required.
Type of effluent produced	The effulent is highly nitrified and stabilized, with BOD in effluent ≤ 20 ppm or so.	The effluent is nitrified up to intrite stage only and is thus less stable, and hence it is slightly inferior quality. BOD in effluent ≥ 30 ppm. Or so.
Doing interval	It generally varies between 3 to 10 minutes. The swage is generally not applied continuously but is applied at intervals.	It is not more than 15 seconds, and the sewage is thus applied continuously.
Filter loading values		
(i) Hydraulic loading	Varies between 20 to 44 M.L. per day	Varies between 11 to 330 M.L. per hectare day.
(ii) Organic loading	Varies between 900 to 2200 kg of BOD_5 per hectare-metre of filter media per day.	Varies between 6000 to 18,000 kg of BOD_5 per hectanre metre of filter media per day.
Recirculation system	Not provided generally.	Always provided for increasing hydraulic loading.
Quality of secondary sludge produced	Black, highly oxidized with slight particles.	Brown, not fully oxidized with fine particles.

Dunbar Filter

Surface loading = 25000 MI/m^2/day.

BOD removed = 85%

Sludge and its Moisture Content

$$\boxed{V = V_1\left[\frac{100 - P_1}{100 - P}\right]}$$

The volume of sludge at moisture content P_1%

The volume of sludge at moisture content P%

Sludge Digestion Tank

(i) When the change during digestion is linear.

(a)

$$\boxed{V = \left(\frac{V_1 + V_2}{2}\right)t}$$

Where,

V = The volume of digestion in m^3.

V_1 = Raw sludge added per day (m^3/day)

Equivalent digested sludge produced per day on completion of digestion, m^3/day.

Digestion period in the day.

(b)

$$\boxed{V = \left(\frac{V_1 + V_2}{2}\right)t + V_2 T}$$

with monsoon storage

Where,

T = Number of days for which digested sludge (V_2) is stored (monsoon) storage)

(ii) When the change during digestion is parabolic

(a)

$$\boxed{V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right]t}$$

without monsoon storage

(b)

$$V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right]t + V_2 T$$

without monsoon storage

Type of raw Sludge to be digested	Kg. of volatile solids present per cu. m. of sludge per month	Per capita capacity in cum/capita
Primary sludge	8	0.021
Mixture of primary sludge and secondary sludge from trickling filter (humus tanks)	7.36	0.036
Mixture of primary sludge and secondary activated sludge	5.76	0.061
Chemically coagulated sludge	-	

Destruction and Removal Efficiency (DRE)

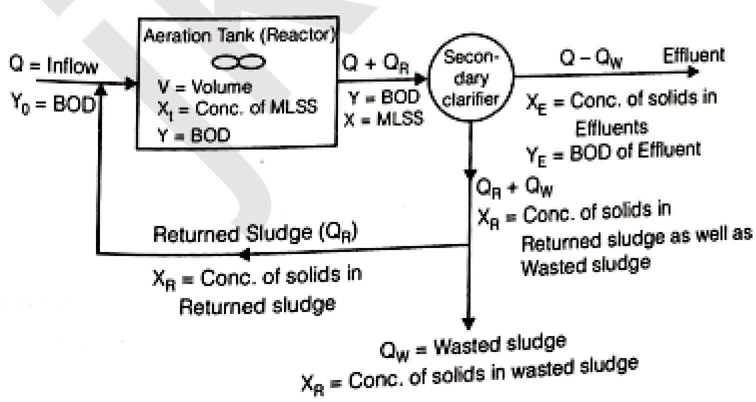
$$DRE = \frac{W_{in} - W_{out}}{W_{in}} \times 100$$

Where,

W_{in} = The mass fill rate of one POHC (Principal organic Hazardous constituent) in the waste stream.

W_{out} = Mass emission rate of the same POHC present in the exhaust emission prior to release to the atmosphere.

Aeration Tank (ASP)



(i) Detention period,

$$\boxed{t = \frac{V}{Q}}$$

Where

V = Volume of the tank in m^3.

Q = Quantity of wastewater flow into the aeration tank excluding the quantity of recycled sludge (m^3/day)

(ii) Volumetric BOD Loading or Organic Loading, (U)

$$\boxed{u = \frac{QY_0}{V}}$$

Where,

$QY_0 =$

Mass of BOD applied per day to the aeration tank through influent sewage in gm.

$V =$ The volume of the aeration tank in m3.

$Q =$ Sewage flows into the aeration tank in m^3.

BOD_5 in mg/lit (or gm/m3) of the influent sewage.

(iii)

$$\boxed{\frac{F}{M} = \frac{QY_o}{VX_t}}$$

Where,

$\frac{F}{M} =$

Food (F) to Microorganism (M) ratio

$QY_o =$

Daily BOD applied to the aeration system in gm.

$Y_o =$

5 day BOD of the influent sewage in mg/lit.

$Q =$

The flow of influent sewage in m^3/day.

MLSS (Mixed liquor suspended solids) in mg/lit.

V = The volume of the Aeration Tank (lit).

$M = X_t V$ = Total microbial mass in the system in gm.

(iv) Sludge Age

(θ_c)

(a)

$$\theta_c = \frac{\text{Mass of suspended solids } (MLSS) \text{ in the system } (M)}{\text{Mass of solids leaving the system per day}}$$

(b)

$$\theta_c = \frac{VX_T}{Q_w X_R + (Q - Q_w) X_E}$$

Where,

X_T = The concentration of solids in the influent of the Aeration Tank called the MLSS i.e. mixed liquor suspended solids in mg/lit.

V = Volume of Aerator

Q_W = The volume of waste sludge per day

The concentration of solids in the returned sludge or in the wasted sludge (both being equal) in mg/lit.

Q = Sewage inflow per day.

X_E = The concentration of solids in the effluent in mg/lit.

(v) Sludge Volume Index (S.V.I)

$$SVI = \frac{V_{ob}}{X_{ob}} \times 1000$$

Where,

X_{ob} = Concentration of suspended solids in the mixed liquor in mg/lit.

V_{ob} = Settled sludge volume in ml/lit.

$S.V.I.$ = Sludge volume index in ml/gm.

(vi) Sludge Recycle and Rate of Return Sludge

$$Q_R \cdot X_R = (Q + Q_R) \times t$$

$$\boxed{\frac{Q_R}{Q} = \frac{X_t}{X_R - X_t}}$$

Where,

Q_R = Sludge recirculation rate in m^3/day.

X_t = MLSS in the aeration tank in mg/lit.

X_R = MLSS in the returned or wasted sludge in mg/lit.

$$\boxed{X_R = \frac{10^6}{S.V.I}}$$

S.V.I = Sludge volume index in ml/gm.

- **Specific substrate utilization rate**

$$\boxed{U = \frac{Q(Y_0 - Y_E)}{V.X_t}} \quad \boxed{\frac{1}{\theta_C} = \alpha_y U - k_e}$$

$\alpha_y = 1$

for MLSS and 0.6 for MLVSS, $k_e = 0.06$

- **Oxygen Requirement of the Aeration Tank**

$$\boxed{O_{2(required)} = \left[\frac{Q(Y_o - Y_E)}{f} - 1.42 Q_W \cdot X_R\right] gm / day}$$

Where,

$$f = \frac{BOD_5}{BOD_u} = \frac{5\,day\,BOD}{Ultimate\,BOD} = 0.68$$

- **Oxygen Transfer Capacity (N)**

$$\boxed{N = \frac{N_S \cdot (D_S - D_L) \cdot (1.024)^{T-20^\circ C} \cdot \alpha}{9.17}}$$

Where,

N = Oxygen transferred under field conditions in kg O_2/k.wh (Or MJ)

N_S = Oxygen transfer capacity under standard conditions in kg O_2/kwh (or MJ)

D_S = Dissolved oxygen-saturation value for sewage at operating temperature.

D_L = Operation D.O level in Areation tank usually 1 to 2 mg/lit.

T = Temperature in ^oC

α = Correction factor for oxygen transfer for sewage usually 0.8 to 0.85.

Oxidation Ponds

- Depth → 1.0 to 1.8 m.
- Detention period → 2 to 6 weeks.
- Organic loading → 150 to 300 kg/ha/day.

 Under hot condition → 60 to 90 kg/ha/day.

 Under cold conditions.

- Length to width ratio = 2
- Sludge Accumulation = 2 to 5 cm/year
- Minimum depth to be kept = 0.3 m.

For Inlet Pipe Design

 Assume V = 0.9 m/s

 Assume flow for 8 hrs.

For Outlet Pipe Design

Dia of outlet = 1.5 dia of the inlet pipe

Septic Tank

- Detention time = 12 to 36 hr.
- Sludge accumulation rate = 30 lit/cap/year.
- Sewage flow = 90 to 150 lit/capita/day.
- Cleaning period = 6 to 12 months
- Length to width ratio = 2 to 3 m.
- Depth = 1.2 to 1.8 m
- Width $\nless$ 0.9 m.
- Free board = 0.3 m.

$$\boxed{\begin{array}{l} Volume\ of\ septic\tan k = \\ (Sewage\,flow \times Detention\,time) \\ +(Sludge\ accumulation\ rate) \times Clearnign\ rate \end{array}}$$